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CHAPTER THIRTY-FIVE

STORAGE FACILITIES

35-1.0 INTRODUCTION

35-1.01 Overview

The traditional design of storm drainage systems has been to collect and convey storm runoff as rapidly as possible to a suitable location where it can be discharged. As areas urbanize, this type of design may result in major drainage and flooding problems downstream. The engineering community is now more conscious of the quality of the environment and the impact that uncontrolled increases in runoff can have on our customers. Under favorable conditions, the temporary storage of some of the storm runoff can decrease downstream flows and often the cost of the downstream conveyance system. This Chapter provides general design criteria for detention/retention storage basins as well as procedures for performing preliminary and final sizing and reservoir routing calculations.

35-1.02 Safety Considerations

Ponding of water for significant periods of time, even at relatively shallow depths, may introduce an additional risk factor for property damage, personal injury or loss of life. All storage facilities in locations easily accessible to the public should be provided with warning signs and fencing adequate to prevent entry onto the site by unauthorized persons. Storage facilities located adjacent to roadways should be provided with an adequate clear zone to minimize the accidental entry of vehicles.

35-1.03 Detention and Retention

Urban storm water storage facilities are often referred to as either detention or retention facilities. For this Chapter, these are defined as follows:

1. Detention. Detention facilities are those that are designed to reduce the peak discharge and only detain runoff for some short period of time. Detention storage involves detaining or slowing runoff and then releasing it. A detention basin has a positive outlet that completely empties all runoff between storms. In some situations, the excavation of a detention facility may extend below the water table or outlet level where the bottom is sealed by sedimentation. This case is referred to as a detention pond or wet bottom

detention basin. The detention pond also has a positive outlet and releases all temporary storage.

Detention facilities may be designed to contain a permanent pool of water. The use of dry bottom detention ponds is strongly recommended for INDOT projects. Because most of the design procedures are the same for wet and dry bottom detention facilities, the term storage facilities will be used in this Chapter to include wet and dry bottom detention facilities.

2. Retention. Retention facilities retain runoff for an indefinite amount of time and have no positive outlet. Runoff is removed only by infiltration through a porous bottom or by evaporation. Retention ponds and lakes are examples of retention facilities that may be built in a development and, in many cases, enhance the overall project. Retention basins are designed to drain into the groundwater table; these are not addressed in this *Manual*.

Most storage facilities will be small in terms of storage capacity and dam height and will serve a single outfall from a watershed of a few hectares. Very small facilities may be contained in parking lots or other on-site facilities. Although the same principals apply to all storage facilities, Section 35-10.0 more specifically relates to the smaller installations.

If special procedures are needed for detention or retention facilities, these will be specified.

35-1.04 Computer Programs

Routing calculations needed to design storage facilities, although not extremely complex, are time consuming and repetitive. To assist with these calculations, there are many available reservoir routing computer programs. When the watershed draining into a storage facility is greater than 0.8 ha, design shall be based upon reservoir routing methods which develop hydrographs for both inflows and outflows. Smaller basins may be analyzed using the storage indication method or the Rational Method.

35-2.0 USES

35-2.01 Introduction

The use of storage facilities for storm water management has increased dramatically in recent years. Controlling the quantity of storm water using storage facilities can provide the following potential benefits.

1. prevention or reduction of peak runoff rate increases caused by urban development,
2. mitigation of downstream drainage capacity problems,

3. reduction or elimination of the need for downstream outfall improvements, and
4. maintenance of historically low flow rates by controlled discharge from storage.

35-2.02 Objectives

The objectives for managing storm water quantity by storage facilities are typically based on limiting peak runoff rates to match one or more of the following values.

1. historic rates for specific design conditions (i.e., post-development peak equals pre-development peak for a particular frequency of occurrence); and

2. non-hazardous discharge capacity of the downstream drainage system.

For a watershed without an adequate outfall, the total volume of runoff is critical and storage facilities are used to store the increases in volume and control discharge rates.

35-3.0 SYMBOLS AND DEFINITIONS

To provide consistency within this Chapter and throughout this *Manual*, the symbols in Figure 35-3A will be used. These symbols were selected because of their wide use in technical publications. In some cases, the same symbol is used in existing publications for more than one definition. Where this occurs in this Chapter, the symbol will be defined where it occurs in the text or equations.

35-4.0 DESIGN CRITERIA

35-4.01 General Criteria

Storage may be developed in depressed areas in parking lots, road embankments, freeway interchanges and small lakes, ponds and depressions within urban developments. The utility of any storage facility depends on the amount of storage, its location within the system, and its operational characteristics. An analysis of such storage facilities should consist of comparing the design flow at a point or points downstream of the proposed storage site with and without storage. In addition to the design flow, other flows in excess of the design flow that might be expected to pass through the storage facility may be required in the analysis (i.e., 100-year flood). The design criteria for storage facilities should include the following:

1. release rate,
2. storage volume,

3. grading and depth requirements,
4. outlet works, and
5. location.

35-4.02 Release Rate

At a minimum, storage facilities shall be designed to detain the 50-year, post-development peak runoff and release it at the 10-year, pre-developed peak runoff rate. If applicable, they shall also meet the more restrictive requirements that may be imposed by a local jurisdiction. An emergency overflow capable of accommodating the 100-year discharge may be required in facilities using a dam.

35-4.03 Storage

Routing calculations must be used to demonstrate that the facility storage volume is adequate to provide the required detention. If sedimentation during construction causes loss of detention volume, design dimensions shall be restored before completion of the project. For detention basins, all detention volume shall be drained within the average period between storm events (72 h).

35-4.04 Grading and Depth

Following is a discussion of the general grading and depth criteria for storage facilities followed by criteria related to wet and dry bottom detention facilities.

35-4.04(01) General

The construction of storage facilities usually requires excavation or placement of earthen embankments to obtain sufficient storage volume. Embankments shall be less than 2 m in height. Vegetated embankments shall have side slopes no steeper than 3H:1V (horizontal to vertical). Riprap-protected embankments shall be no steeper than 2H:1V. Excavated storage facilities shall not have an operating design pool depth greater than 1.5 m unless specifically approved by the INDOT Hydraulics Engineer.

A minimum freeboard of 0.3 m above the 100-year storm, high-water elevation shall be provided.

Other considerations when setting depths include flood elevation requirements, public safety, land availability, land value, present and future land use, water table fluctuations, soil characteristics, maintenance requirements and required freeboard. Aesthetically pleasing features are also important in urbanizing areas. Fencing of basins is addressed in Section 35-14.0.

35-4.04(02) Dry Bottom Detention

Areas above the normal high-water elevations of storage facilities should be sloped toward the facilities to allow drainage and to prevent standing water. Careful finish grading is required to avoid creation of upland surface depressions that may retain runoff. The bottom area of storage facilities should be graded toward the outlet to prevent standing water conditions. A low flow or pilot channel constructed across the facility bottom from the inlet to the outlet is recommended to convey low flows and prevent standing water conditions.

35-4.04(03) Wet Bottom Detention

The maximum depth of permanent storage facilities will be determined by site conditions and design constraints. In general, if the facility provides a permanent pool of water, a depth sufficient to discourage growth of weeds should be considered. A depth of 2.0 m is generally reasonable.

35-4.05 Outlet Works

Outlet works selected for storage facilities typically include a principal spillway and an emergency overflow and must be able to accomplish the design functions of the facility. Outlet works can take the form of combinations of drop inlets, pipes, weirs and orifices. Slotted riser pipes are discouraged because of clogging problems, but curb openings may be used for parking lot storage. The principal spillway is intended to convey the design storm without allowing flow to enter an emergency outlet.

Orifice outlets commonly take the form of restrictions (300 mm or less) placed in larger pipes. The preferred design for such outlets consists of placing a smaller pipe on the flowline of a larger pipe. The smaller pipe will be the required size to achieve the desired detention results and would be approximately 300 mm in length. Grout is placed around the smaller pipe to fill the area of the larger. This type of construction provides for adequate maintenance and is more durable than a single constrictor plate.

35-5.0 GENERAL PROCEDURE

35-5.01 Data Needs

The following data will be needed to complete storage design and routing calculations.

1. inflow hydrograph for all selected design storms;

2. stage-storage curve for proposed storage facility (see Figure 35-5A for an example). For large storage volumes, use hectare/meters; otherwise use cubic meters; and

3. stage-discharge curve for all outlet control structures (see Figure 35-5B for an example).

Using these data, a design procedure is used to route the inflow hydrograph through the storage facility with different basin and outlet configurations until the desired outflow hydrograph is achieved. See the example problem in Section 35-8.0.

35-5.02 Stage-Storage Curve

A stage-storage curve defines the relationship between the depth of water and storage volume in a reservoir. The data for this type of curve are usually developed using a topographic map and one of the following formulas □ the average-end area, frustum of a pyramid or prismoidal formulas. Storage basins are often irregular in shape to blend well with the surrounding terrain and to improve aesthetics. Therefore, the average-end area formula is usually preferred as the method to be used on non-geometric areas. The double-end area formula is expressed as follows:

$$V_{1,2} = [(A_1 + A_2)/2]d \qquad \text{(Equation 35-5.1)}$$

Where:

$V_{1,2}$ = storage volume, m^3, between elevations 1 and 2
A_1 = surface area at elevation 1, m^2
A_2 = surface area at elevation 2, m^2
d = change in elevation between points 1 and 2, m

The frustum of a pyramid is expressed as follows:

$$V = d/3\ [A_1 + (A_1A_2)^{0.5} + A_2] \qquad \text{(Equation 35-5.2)}$$

Where:

V = volume of frustum of a pyramid, m^3
d = change in elevation between points 1 and 2, m
A_1 = surface area at elevation 1, m^2
A_2 = surface area at elevation 2, m^2

The prismoidal formula for trapezoidal basins is expressed as follows:

$$V = LWD + \frac{(L + W)\,D^2}{Z} + \frac{4\,D^3}{3\,Z^2} \qquad \text{(Equation 35-5.3)}$$

Where:

V	=	volume of trapezoidal basin, m^3
L	=	length of basin at base, m
W	=	width of basin at base, m
D	=	depth of basin, m
Z	=	side slope factor, ratio of horizontal to vertical

35-5.03 Stage-Discharge Curve

A stage-discharge curve defines the relationship between the depth of water and the discharge or outflow from a storage facility. A typical storage facility has two spillways: principal and emergency. The principal spillway is usually designed with a capacity sufficient to convey the design flood without allowing flow to enter the emergency spillway. A pipe culvert, weir or other appropriate outlet can be used for the principal spillway or outlet. Tailwater influences and structure losses must be considered when developing discharge curves.

The emergency spillway, when needed, is sized to provide a bypass for floodwater during a flood that exceeds the design capacity of the principal spillway. This spillway should be designed taking into account the potential threat to downstream life and property if the storage facility were to fail. The stage-discharge curve should take into account the discharge characteristics of both the principal and emergency spillways.

35-5.04 General Procedure

A general procedure for using the above data in the design of storage facilities is presented below.

1. Compute inflow hydrographs for runoff from the design storm using the procedures outlined in Chapter Twenty-nine.

2. Perform preliminary calculations to evaluate detention storage requirements for the hydrographs from Step 1 (see Section 35-7.0 for recommended methods).

3. Determine the physical dimensions necessary to hold the estimated volume from Step 2, including freeboard. The maximum storage requirement calculated from Step 2 should be used.

4. Size the outlet structure. The estimated peak stage will occur for the estimated volume from Step 2. The outlet structure should be sized to convey the allowable discharge at this stage. Ascertain that tailwater effects have been considered.

5. Perform routing calculations using inflow hydrographs from Step 1 to check the preliminary design using the storage routing equations. If the routed post-development peak discharges from the 50-year design storm exceeds the pre-development 10-year peak discharge, or if the peak stage varies significantly from the estimated peak stage from Step 4, then revise the estimated volume and return to Step 3.

6. Where required, consider emergency overflow from runoff due to the 100-year design storm and established freeboard requirements.

7. Evaluate the control structure outlet velocity and provide channel and bank stabilization if the velocity will cause erosion problems downstream.

This procedure can involve a significant number of reservoir routing calculations to obtain the desired results.

35-5.05 Computer Procedures

A number of commercial computer software packages exist which automate several of the steps described above. Although these programs can greatly accelerate the design process, they should be used with caution, and the output from these programs should be critically reviewed considering sound engineering judgment. Except when modeling drainage areas less than 0.8 ha, the programs must be capable of developing hydrographs for both inflows and outflows. For areas less than 0.8 ha, the Rational Method is acceptable for generating the inflow hydrographs.

35-6.0 OUTLET HYDRAULICS

35-6.01 Outlets

Sharp-crested weir flow equations for no-end contractions, two-end contractions and submerged discharge conditions are presented below, followed by equations for broad-crested weirs, v-notch weirs, proportional weirs, orifices or combinations of these facilities. If culverts are used as outlet works, procedures presented in Chapter Thirty-one should be used to develop stage-discharge data. When analyzing release rates, tailwater influences must be considered to determine the effective head on each outlet. Slotted riser pipe outlet facilities should be avoided.

35-6.02 Sharp-Crested Weirs

A sharp-crested weir with no-end contractions is illustrated in Figure 35-6A. The discharge equation for this configuration is as follows (Chow, 1959):

$$Q = [(1.805 + 0.221(H/H_c)] LH^{1.5} \qquad \text{(Equation 35-6.1)}$$

Where:
- Q = discharge, m^3/s
- H = head above weir crest excluding velocity head, m
- H_c = height of weir crest above channel bottom, m
- L = horizontal weir length, m

A sharp-crested weir with two-end contractions is illustrated in Figure 35-6B. The discharge equation for this configuration is as follows (Chow, 1959):

$$Q = [(1.805 + 0.221(H/H_c)] (L - 0.2H) H^{1.5} \qquad \text{(Equation 35-6.2)}$$

Where: Variables are the same as Equation 35-6.4.

Click here to view Figure 35-6C, a sharp-crested weir and head.

A sharp-crested weir will be affected by submergence when the tailwater rises above the weir crest elevation. The result will be that the discharge over the weir will be reduced. The discharge equation for a sharp-crested submerged weir is as follows (Brater and King, 1976):

$$Q_S = Q_f(1 - (H_2/H_1)^{1.5})^{0.385} \qquad \text{(Equation 35-6.3)}$$

Where:
- Q_s = submergence flow, m^3/s
- Q_f = free flow, m^3/s
- H_1 = upstream head above crest, m
- H_2 = downstream head above crest, m

35-6.03 Broad-Crested Weirs

The equation generally used for the broad-crested weir is as follows (Brater and King, 1976):

$$Q = CLH^{1.5} \qquad \text{(Equation 35-6.4)}$$

Where: Q = discharge, m^3/s
C = broad-crested weir coefficient
L = broad-crested weir length, m
H = head above weir crest, m

If the upstream edge of a broad-crested weir is so rounded as to prevent contraction and if the slope of the crest is as great as the loss of head due to friction, flow will pass through critical depth at the weir crest; this yields the maximum C value of 1.704. For sharp corners on the broad-crested weir, a minimum C value of 1.435 should be used. Additional information on C values as a function of weir crest breadth and head is given in Figure 35-6D.

35-6.04 V-Notch Weirs

The discharge through a V-notch weir can be calculated from the following equation (Brater and King, 1976).

$$Q = 1.38 \tan(\varphi/2)H^{2.5} \qquad \text{(Equation 35-6.5)}$$

Where: Q = discharge, m^3/s
φ = angle of V-notch, degrees
H = head on apex of notch, m

35-6.05 Proportional Weirs

Although more complex to design and construct, a proportional weir may significantly reduce the required storage volume for a given site. The proportional weir is distinguished from other control devices by having a linear head-discharge relationship achieved by allowing the discharge area to vary nonlinearly with head.

Design equations for proportional weirs are as follows (Sandvik, 1985):

$$Q = 2.74\, a^{0.5}\, b(H - a/3) \qquad \text{(Equation 35-6.6)}$$
$$x/b = 1 - (1/3.17)\, (\arctan\, (y/a)^{0.5}) \qquad \text{(Equation 35-6.7)}$$

Where: Q = discharge, m^3/s

Dimensions a, b, H, x, and y are shown in Figure 35-6E.

35-6.06 Orifices

Pipes smaller than 300 mm may be analyzed as a submerged orifice if H/D is greater than 1.5. For square-edged entrance conditions, the formula applies as follows:

$$Q = 0.6A(2gH)^{0.5} \qquad \text{(Equation 35-6.8)}$$

Where:

Q = discharge, m^3/s

A = cross-section area of pipe, m^2

g = acceleration due to gravity, 9.81 m/s^2

D = diameter of pipe, m

H = head on pipe, from the center of pipe to the water surface, m*

**Note:* *In cases where the tailwater is higher than the center of the opening, the head is calculated as the difference in water surface elevations.*

35-7.0 PRELIMINARY DETENTION CALCULATIONS

35-7.01 Storage Volume

A preliminary estimate of the storage volume required for peak flow attenuation may be obtained from a simplified design procedure that replaces the actual inflow and outflow hydrographs with the standard triangular shapes shown in Figure 35-7A.

The required storage volume may be estimated from the area above the outflow hydrograph and inside the inflow hydrograph, expressed as follows:

$$V_S = 0.5T_i(Q_i - Q_o) \qquad \text{(Equation 35-7.1)}$$

Where:

V_S = storage volume estimate, m^3

Q_i = peak inflow rate, m^3/s

Q_o = peak outflow rate, m^3/s

T_i = duration of basin inflow, s

Any consistent units may be used for Equation 35-7.1.

35-7.02 Alternative Method

An alternative preliminary estimate of the storage volume required for a specified peak flow reduction can be obtained by the following regression equation procedure (Wycoff & Singh, 1986).

1. Determine input data, including the allowable peak outflow rate, Q_o, the peak flow rate of the inflow hydrograph, Q_i, the time base of the inflow hydrograph, t_b, and the time to peak of the inflow hydrograph, t_p.

2. Calculate a preliminary estimate of the ratio V_S/V_r using the input data from Step 1 and the following equation:

$$V_S/V_r = [1.291(1 - Q_o/Q_i)^{0.753}]/[(t_b/t_p)^{0.411}] \quad \text{(Equation 35-7.2)}$$

Where:
- V_S = volume of storage, m^3
- V_r = volume of runoff, m^3
- Q_o = outflow peak flow, m^3/s
- Q_i = inflow peak flow, m^3/s
- t_b = time base of the inflow hydrograph, h
  (Determined as the time from the beginning of rise to a point on the recession limb where the flow is 5% of the peak.)
- t_p = time to peak of the inflow hydrograph, h

35-7.03 Peak Flow Reduction

A preliminary estimate of the potential peak flow reduction for a selected storage volume can be obtained by the following procedure.

1. Determine the following:

   a. volume of runoff, V_r
   b. peak flow rate of the inflow hydrograph, Q_i
   c. time base of the inflow hydrograph, t_b
   d. time to peak of the inflow hydrograph, t_p
   e. storage volume, V_S

2. Calculate a preliminary estimate of the potential peak flow reduction for the selected storage volume using the following equation (Singh, 1976).

$$Q_o/Q_i = 1 - 0.712(V_S/V_r)^{1.328}(t_b/t_p)^{0.546} \quad \text{(Equation 35-7.3)}$$

Where: Q_o = outflow peak flow, m^3/s

Q_i	=	inflow peak flow, m^3/s
V_S	=	volume of storage, m^3
V_r	=	volume of runoff, m^3
t_b	=	time base of the inflow hydrograph, h (Determined as the time from the beginning of rise to a point on the recession limb where the flow is 5% of the peak.)
t_p	=	time to peak of the inflow hydrograph, h

3. Multiply the peak flow rate of the inflow hydrograph, Q_i, times the potential peak flow reduction calculated from Step 2 to obtain the estimated peak outflow rate, Q_o, for the selected storage volume.

35-7.04 Preliminary Basin Dimensions

The following applies.

1. Plot the control structure location on a contour map.

2. Select a desired depth of ponding for the design storm.

3. Divide the estimated storage volume needed by the desired depth to obtain the surface area required of the reservoir.

4. Based on site conditions and contours, estimate the geometric shape(s) required to provide the estimated reservoir surface area.

35-8.0 ROUTING CALCULATIONS

The following procedure is used to perform routing through a reservoir or storage facility (Puls Method of storage routing).

1. Develop an inflow hydrograph, stage-discharge curve and stage-storage curve for the proposed storage facility. Example stage-storage and stage-discharge curves are shown in Figures 35-8A and 35-8B, respectively.

2. Select a routing time period, Δt, to provide at least five points on the rising limb of the inflow hydrograph ($\Delta t < T_c/5$).

3. Use the storage-discharge data from Step 1 to develop storage characteristics curves that provide values of $SK(O_1/2)\Delta t$ versus stage. An example tabulation of storage characteristics curve data is shown in Figure 35-8C.

4. For a given time interval, I_1 and I_2 are known. Given the depth of storage or stage, H_1, at the beginning of that time interval, $S_1 - (O_1/2)\Delta t$ can be determined from the appropriate storage characteristics curve (example given in Figure 35-8D).

5. Determine the value of $S_2 + (O_2/2)\Delta t$ from the following equation.

$$S_2 + (O_2/2)\Delta t = [S_1 - (O_1/2)\Delta t] + [(I_1 + I_2)/2]\Delta t \qquad \text{(Equation 35-8.1)}$$

Where:

	S_2	=	storage volume at time 2, m^3
	O_2	=	outflow rate at time 2, m^3/s
	Δt	=	routing time period, s
	S_1	=	storage volume at time 1, m^3
	O_1	=	outflow rate at time 1, m^3/s
	I_1	=	inflow rate at time 1, m^3/s
	I_2	=	inflow rate at time 2, m^3/s

Other consistent units are equally appropriate.

6. Enter the storage characteristics curve at the calculated value of $S_2 + (O_2/2)\Delta t$ determined in Step 5 and read off a new depth of water, H_2.

7. Determine the value of O_2, which corresponds to a stage of H_2 determined in Step 6, using the stage-discharge curve.

8. Repeat Steps 1 through 7 by setting new values of I_1, O_1, S_1 and H_1 equal to the previous I_2, O_2, S_2 and H_2, and using a new I_2 value. This process is continued until the entire inflow hydrograph has been routed through the storage basin.

35-9.0 EXAMPLE PROBLEM

35-9.01 Example

This example demonstrates the application of the methodology presented in this Chapter for the design of a typical detention storage facility. Example inflow hydrographs and associated peak discharges for both pre- and post-development conditions are assumed to have been developed using hydrologic methods from Chapter Twenty-nine.

35-9.02 Design Discharge and Hydrographs

Storage facilities are to be designed for runoff from both the 10-year and 50-year design storms. INDOT requires that the 50-year post-development peak discharge meet or not exceed the 10-year pre-development peak discharge. Example peak discharges from the 10-year and 50-year design storm events are as follows:

1. Pre-development 10-year peak discharge = 5.66 m^3/s
2. Post-development 50-year peak discharge = 7.08 m^3/s

Because the post-development 50-year peak discharge must not exceed the pre-development 10-year peak discharge, the allowable outflow discharge cannot exceed 5.66 m^3/s.

Example runoff hydrographs are shown in Figure 35-9A. Inflow durations from the post-development hydrographs are approximately 1.2 and 1.25 h, respectively, for runoff from the 10-year and 50-year storms.

35-9.03 Preliminary Volume Calculations

Preliminary estimates of required storage volumes are obtained using the simplified method outlined in Section 35-7.0. The required storage volume, V_S, to contain the difference between $Q_{50\ (POST)}$ and $Q_{10\ (PRE)}$ is computed using Equation 35-7.1.

$$V_S = 0.5T_i(Q_i - Q_o) = 0.5T_i\ (Q_{50(POST)} - Q_{10(PRE)})$$
$$V_S = 0.5\ (1.25)\ (3600)\ (7.08 - 5.66) = 3195\ m^3$$

35-9.04 Design and Routing Calculations

Stage-discharge and stage-storage characteristics of a storage facility that should provide adequate peak flow attenuation for runoff from the 50-year design storm is presented in Figure 35-9B. The storage-discharge relationship was developed by requiring the preliminary storage volume estimates of runoff for the 50-year design storm to be provided when the corresponding allowable peak discharge occurred. Discharge values were computed by solving the broad-crested weir equation for head, H, assuming a constant discharge coefficient of 1.71, a weir length of 1.22 m and no tailwater submergence. The capacity of storage relief structures was assumed to be negligible.

Storage routing was conducted for runoff from the 50-year design storm to confirm the preliminary storage volume estimates and to establish design water surface elevations. Routing results using the Stage-Discharge-Storage data given in Figure 35-9B and the Storage Characteristics Curves given on Figures 35-8A and 35-8B, and 0.1-h time steps are shown in Figure 35-9C for runoff from the 50-year design storm. The preliminary design provides adequate peak discharge attenuation.

For the routing calculations, the following equation was used:

$$S_2 + (O_2/2)\Delta t = [S_1 - (O_1/2)\Delta t] + [(I_1 + I_2)/2\Delta t] \qquad \text{(Equation 35-9.1)}$$

Also, Column 6 = Column 3 + Column 5.

Because the routed peak discharge is lower than the maximum allowable peak discharges for the 50-year design storm event, the weir length could be increased or the storage decreased. If revisions are desired, routing calculations must be repeated.

Although not shown for this example, runoff from the 100-year storm should be routed through the storage facility to establish freeboard requirements and to evaluate emergency overflow and stability requirements. In addition, the preliminary design provides hydraulic details only. Final design should consider site constraints such as depth of water, side slope stability and maintenance, grading to prevent standing water, and provisions for public safety.

35-10.0 DRY BOTTOM DETENTION BASIN

35-10.01 Introduction

Dry bottom detention basins are depressed areas that store runoff during wet weather and are dry the rest of the time. They are very popular because of their comparatively low cost; few design limitations; and ability to serve large and small watersheds.

35-10.02 Design

The following applies.

1. Quantity. The pond should be designed to provide the required detention, and it should be able to pass a 100-year storm safely. It should be designed using the procedures described in Sections 35-5.4 and 35-5.5, and a 100-year storm should be routed through the facility to ensure that the embankment will not be damaged or fail during the passage of that storm. To improve the efficiency of the outlet, it may be necessary to include an antivortex device.

2. Outlets. Outlets for dry basins can be designed in a wide variety of configurations, although INDOT discourages the use of riser pipes. Larger flows are usually accommodated by an emergency spillway.

35-10.03 Other Considerations

The side slopes of the pond should be no steeper than 3H:1V to facilitate maintenance activities. In addition, the floor of the pond should be sloped at 2% toward the outlet to prevent ponding. Maximum operating pool depth should not exceed 2.0 m without specific approval of the INDOT Hydraulics Engineer.

Routine maintenance activities include an annual inspection (preferably during wet weather) and mowing, as needed.

35-11.0 WET BOTTOM DETENTION BASIN

35-11.01 Introduction

A wet bottom detention basin is very similar to a dry bottom detention basin in that it detains stormwater, but it is different in that it retains a permanent pool during dry weather. Wet bottom detention basins are usually more expensive than dry bottom detention basins.

35-11.02 Design

The following applies.

1. Quantity. A wet bottom detention basin should provide the required detention and be able to pass a 100-year storm safely.

2. Outlet. Outlets for wet ponds can be designed in a wide variety of configurations, but INDOT discourages the use of riser pipes.

35-11.03 Other Considerations

The side slopes of the pond should be no steeper than 3H:1V both above and below water for both safety and maintenance. Normal pool depth should not exceed 1.5 m and the maximum operating pool depth should not exceed 2.5 m.

Routine maintenance includes annual inspections, preferably during wet weather and mowing as needed.

35-11.04 Illustration

See Figure 35-11A for an illustration of a typical wet pond.

35-12.0 LAND-LOCKED RETENTION

Watershed areas that drain to a central depression with no positive outlet are typical of many topographic areas including karst topography, and they can be evaluated using a mass flow routing procedure to estimate flood elevations. Although this procedure is fairly straightforward, the evaluation of basin outflow is a complex hydrogeologic phenomenon that requires good field measurements and a thorough understanding of local conditions. Because outflow rates for flooded conditions are difficult to calculate, field measurements are desirable. The designer is referred to the AASHTO *Model Drainage Manual* for more detailed information.

35-13.0 CONSTRUCTION AND MAINTENANCE CONSIDERATIONS

An important step in the design process is identifying whether special provisions are warranted to properly construct or maintain proposed storage facilities. To ensure acceptable performance and function, storage facilities that require extensive maintenance are discouraged. The following maintenance problems are typical of urban detention facilities.

1. weed growth,
2. grass and vegetation maintenance,
3. bank deterioration,
4. standing water or soggy surfaces,
5. mosquito control,
6. blockage of outlet structures,
7. litter accumulation, and
8. maintenance of fences and perimeter plantings.

Proper design should focus on the elimination or reduction of maintenance requirements by addressing the potential for problems to develop the following:

1. Both weed growth and grass maintenance may be addressed by constructing side slopes that can be maintained using available power-driven equipment, such as tractor mowers.

2. Bank deterioration can be controlled with protective lining or by limiting bank slopes.

3. Standing water or soggy surfaces may be eliminated by sloping basin bottoms toward the outlet, constructing low-flow pilot channels across basin bottoms from the inlet to the outlet.

4. In general, when the above problems are addressed, mosquito control will not be a major problem.

5. Outlet structures should be selected to minimize the possibility of blockage (i.e., very small pipes tend to block quite easily and should be avoided).

6. Finally, one way to deal with the maintenance associated with litter and damage to fences and perimeter plantings is to locate the facility for easy access where this maintenance can be conducted on a regular basis.

35-14.0 PROTECTIVE TREATMENT

Safety considerations include reducing the chance of drowning by fencing the basin, reducing the maximum depth, and/or including ledges and mild slopes to prevent people from falling in and facilitate their escape from the basin. Protective treatment may be required to prevent entry to facilities that present a hazard to children and, to a lesser extent, all persons. Fences and signs will be required for detention areas where one or more of the following conditions exist.

1. Rapid stage increases would make escape difficult.
2. Water depths either exceed 1 m for more than 24 h or are permanently wet.
3. Side slopes equal or exceed 1.5H:1V.

Where storage facilities are located near a roadway, the road should be provided with an adequate clear zone. Finally, maximum operating pool depths will be limited to 2.0 m unless specifically approved by the INDOT Hydraulics Engineer.

35-15.0 REFERENCES

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